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THE EFFECT OF RESIDUAL LONGITUDINAL STRESSES UPON THE LOAD-CARRYING CAPACITY OF STEEL COLUMNS

A REPORT OF AN INVESTIGATION

CONDUCTED BY

THE ENGINEERING EXPERIMENT STATION
UNIVERSITY OF ILLINOIS

IN COÖPERATION WITH

THE NICKEL PLATE ROAD

BY

WILBUR M. WILSON

AND

REX L. BROWN

BULLETIN No. 280
ENGINEERING EXPERIMENT STATION
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ENGINEERING EXPERIMENT STATION

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THE EFFECT OF RESIDUAL LONGITUDINAL STRESSES UPON THE LOAD-CARRYING CAPACITY OF STEEL COLUMNS

I. Introduction

1. *Object of Tests.*—Many steel bridges have become inadequate in strength not so much because the bridge has deteriorated, but because the loads which they are required to carry are much greater than the load for which they were designed. A convenient method that has been used to strengthen these bridges is to increase the section of members by welding longitudinal steel plates or shapes to the members while they remained in place on the bridge. It is a matter of common experience that when a longitudinal plate or shape is welded to a structural member, longitudinal thermal stresses are produced which may have considerable magnitude. These stresses are due to the large variation in temperature that occurs over a transverse section immediately after a weld is made. There is a narrow longitudinal strip adjacent to the welds that is hot, whereas the steel some distance from the welds is comparatively cool. After the welds have been completed, the whole member gradually cools to the temperature of the surrounding air. All parts are cooled, but some more than others. Portions that cool the most shrink the most, and this difference in shrinkage causes residual longitudinal stresses in the steel, tension at some portions of a section and compression at others. The question of importance to the structural engineer is, Do these residual longitudinal stresses affect the load-carrying capacity of a steel column? The tests described in the following pages were planned to give information relative to this question.

2. *Acknowledgments.*—The tests described in this report were a part of the research work of the Engineering Experiment Station of the University of Illinois, of which Dean M. L. Enger is the Director, and of the Department of Civil Engineering, of which Prof. W. C. Huntington is the head. The work was planned in consultation with an Advisory Committee, representing the coöperators, which consisted of the following members: George H. Tinker, Bridge Engineer of the Nickel Plate Road, Chairman; A. C. Harvey, J. C. Wallace, and F. S. Hales, Chief Engineer, Assistant Chief Engineer, and Engineer of Track, respectively, of the Nickel Plate Road; Jonathan Jones,

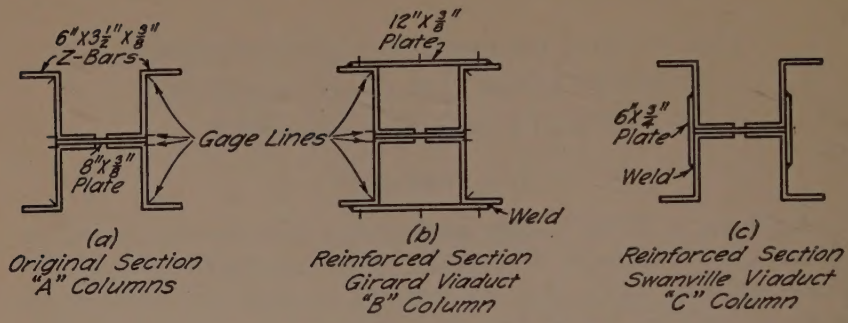


FIG. 1. SECTION OF STEEL VIADUCT COLUMNS

Chief Engineer, McClintic-Marshall Corporation; and ALBERT REICHMANN, Assistant Chief Engineer, American Bridge Company.

The expenses of the tests were paid from funds furnished by the Nickel Plate Road; the steel was furnished by the American Bridge Company; the columns were fabricated by the McClintic-Marshall Corporation; the columns were transported to the laboratory by the Illinois Central Railroad; and the welding rods were furnished by the Lincoln Electric Company and the Una Welding Company of Cleveland, Ohio.

The authors were assisted in the experimental work by RALPH W. KLUGE, F. B. METTERHAUSEN, and FRANK P. THOMAS.

II. FIELD OBSERVATIONS ON THE GIRARD VIADUCT

3. *Description of Girard Viaduct.*—The Nickel Plate Road is carried over a ravine about three miles west of Girard, Pa., on a steel viaduct which was built in 1902, and which has been in continuous service since that date. During this period the weight of the heaviest rolling stock for the road has increased approximately 50 per cent. Because of this increase in load, the viaduct was reinforced in 1934 by welding longitudinal steel plates on the steel columns, and adding laterals so as to shorten the unsupported length of the columns. The original section of the columns consisted of a web plate and four Z bars as shown in Fig. 1a. A viaduct at Swanville, which had the same column section as the Girard viaduct, had been reinforced by adding two 6 in. by ¾ in. plates as shown in Fig. 1c but, because for portions of some columns the web plate projected beyond the web of the Z bars, and because the webs of adjacent Z bars were not always in the same plane, some difficulty had been encountered in adding the plates,

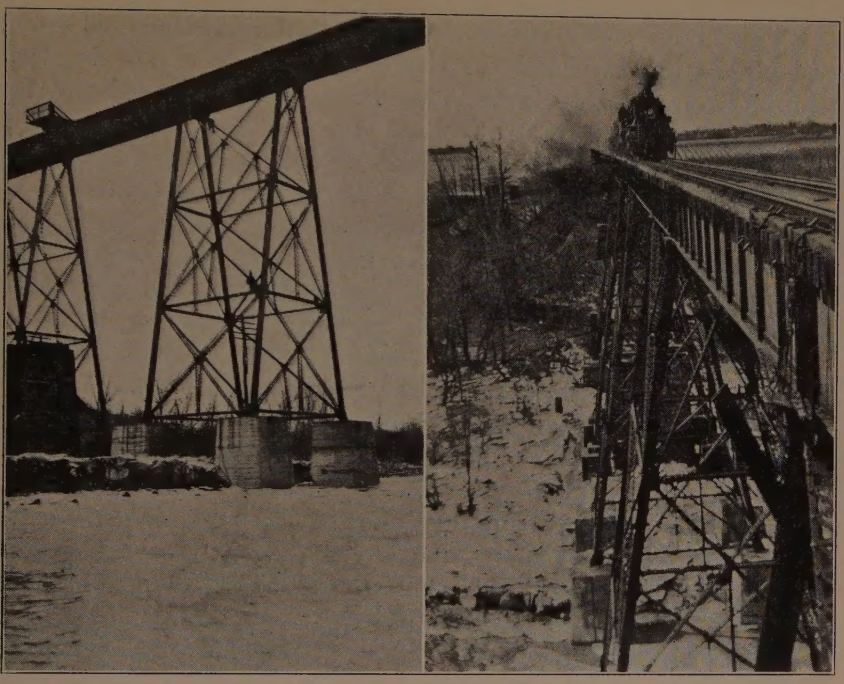


FIG. 2. GENERAL VIEW OF GIRARD VIADUCT AT TIME IT WAS BEING REINFORCED

and the reinforcement shown in Fig. 1b was used for the Girard viaduct. The increase in section is about 50 per cent.

Figure 2 shows two views of the viaduct. The view at the left shows a tower before the reinforcement was added, and the one to the right a tower after it had been reinforced. The horizontal struts, which were added to the lateral bracing to reduce the unsupported length of the columns, can be seen in the latter view. The total length of the viaduct is about 1500 ft. and the height of the steel towers over the central portion is approximately 85 ft. There is a 30-ft. deck-girder span over each tower and a 60-ft. deck-girder span connecting adjacent towers. Observations were made on two columns before and after they were reinforced, as described in Section 4.

4. *Field Observations.*—The objects of making the observations were (1) to determine the temperature deformations induced in the columns of the viaduct by the process of welding reinforcing plates on the Z-bar columns; (2) to determine the distribution of the live-load stress over the section of the column, for both the original and the reinforced columns.

Observations were made on column 20 north and column 20 south, two columns of a bent near the middle of the viaduct. The live-load strain was measured at a section near the top of the top panel only for the original section; both the live-load strain on the reinforced section and the temperature deformation due to welding were measured at sections near the top of the top panel, near the bottom of the top panel, and near the bottom of the bottom panel for both columns. The deformation was measured with an 8-in. Berry strain gage on a number of gage lines at each section of each column. The location of these gage lines is indicated on the section of the column shown in Fig. 1. Steel standard bars were clamped to the columns at points adjacent to the gage lines so that the bars would have the same exposure to sun and wind as the steel in the columns whose deformation was being measured.

In determining the temperature deformation due to welding, a complete set of strain readings was taken on both the original section and on the reinforcing plate after the latter had been clamped in place but before welding had begun. A second set of readings was taken after the plate had been completely welded in place and after it had cooled to the temperature of the air. The difference between two readings on a gage line, one taken before and the other after welding, was considered as representing the deformation due to welding.

In determining the live-load stress, a complete set of readings was taken when the bridge carried no load except its own weight. A locomotive was then run upon the viaduct and brought to rest, headed west, with the second driver over the columns on which observations were being made. A second set of readings was then recorded. The difference between two readings, one taken with the engine off and the other with the engine on the viaduct, was considered as representing the strain produced by the weight of the engine. In all instances, readings were taken in duplicate, and the allowable difference between two readings that were supposed to check was one division on the dial, which corresponds to the elastic strain due to a stress of 750 lb. per sq. in. No difficulty was experienced in meeting this tolerance requirement, and all checks that were applied to the data indicate that the deformations were read as accurately on the viaduct as they are usually read in the laboratory.

5. *Live-Load Stresses.*—The engine, when headed west and placed with the second driver over column 20, produced a total load on each column of approximately 149 000 lb., or an average stress of 7000 lb.

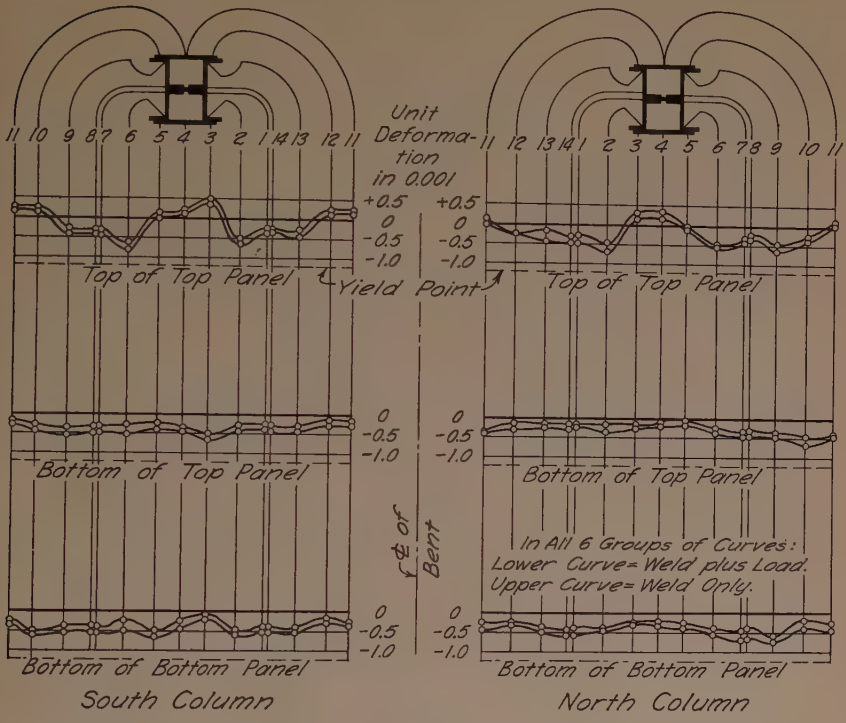


FIG. 3. UNIT DEFORMATION DUE TO WELDING AND TO LOAD; GIRARD VIADUCT

per sq. in. in the original column and 4900 lb. per sq. in. in the reinforced column. The average live-load stress at the top of the top panel of the original column, as determined from the measured strain on the basis that $E = 30\ 000\ 000$ lb. per sq. in., is 8200 lb. per sq. in. for the north column and 7980 lb. per sq. in. for the south column. The average values of the measured live-load stress in the reinforced columns (1) at the top of the top panel, (2) at the bottom of the top panel, and (3) at the bottom of the bottom panel, are 4420, 3860, and 4690 lb. per sq. in., respectively, for the north column, and 3700, 5190, and 4300 lb. per sq. in., respectively, for the south column. The average of the values for the corresponding sections of the two columns are 4060, 4525, and 4495 lb. per sq. in. for the sections at the top of the top panel, at the bottom of the top panel, and at the bottom of the bottom panel, respectively. The distribution of the live-load strain over the sections of the reinforced columns is shown in Fig. 3. The vertical distance between the two curves represents the measured deformation due to the weight of the engine.

The average measured stress on the original columns is somewhat more than the average value computed from the weight of the engine, whereas, for the reinforced columns, the average of measured values is somewhat less than the computed value. The discrepancy is 16 per cent for the original column and 11 per cent for the reinforced one. These discrepancies, which are not especially large for values obtained as these have been, might be caused in part by errors due to temperature changes, but the use of several standard bars distributed so that the bar controlling a given gage line would have the same sun and wind exposure as the steel being tested should keep the errors due to temperature changes small. The discrepancy was more probably due to the fact that the stress was not the same at other points on a section as it was at the points where the strain was measured, and the average of the values at the gage lines was not the average for the whole section.

The distribution of the live-load stress is, in general, as uniform for the reinforced column as it is for the original section. Moreover, the distribution of the live-load stress on these columns in the field structure is about as uniform as the distribution over a section of a built-up column when loaded in a testing machine.

6. *Thermal Deformation.*—The deformation due to welding, shown in Fig. 3, is a shortening at all points where measurements were taken for the section at the bottom of the top panel and the bottom of the bottom panel for both columns, but there is an elongation of the reinforcing plates at some points at the top of the top panel. The shortening at all points of a section is explained as follows:

When the edge of the plate is welded, some steel near the corners of the column is fused and adjacent portions are heated to a cherry red. The rise in temperature drops off rapidly with the distance from the welds and the temperature of the steel is raised comparatively little above the temperature of the air at gage lines 4, 1, 14 and other corresponding points. The temperature at the corners of the column was so high that the paint peeled off at gage lines 2, 3 and other corresponding points. The temperature deformation due to a temperature change of 200 deg. F. is equal to the elastic deformation due to a stress of 38 000 lb. per. sq. in. With the steel at gage lines 4, 1 and 14 comparatively cool, the thermal expansion at gage lines 2 and 3, when this portion of the section is heated, is resisted and plastic flow (shortening) takes place. When the column cools, the shrinkage at 2 and 3 is prevented in part by the adjacent metal which has remained

comparatively cool. The result is that the final stress at 2 and 3 is tension but, because of the plastic shortening that took place at 2 and 3 when the weld was made, the distance between the gage holes is less than it was before the weld was begun. That is, although the strain gage shows a shortening over the entire area for many sections, there is probably tensile stress at the corners of the section near the weld and compressive stress at points some distance from the weld. The fact should be borne in mind, therefore, in studying the diagrams of Fig. 3, that the deformation may not indicate either the magnitude or sense of the stress. It is believed, however, that the deformation at gage lines 4, 7, 8, 11, 14, and 1 represent stress, but the deformation given for gage lines 2 and 3, 5 and 6, 9 and 10, and 12 and 13 are partly plastic flow and partly elastic deformation due to stress. The shortening of a column due to welding was so great that it could be detected by measuring its overall length with a steel tape, one measurement being taken before the welding was begun and the other after it was completed.

The deformation at the top of the top panel is partly elongation and partly contraction. These gage lines were near the upper end of the upper plate (the upper hole of the 8-in. gage line was only 5 inches from the extreme end of the plate) and the temperature deformation on the section may have been affected by the butt weld across the end of the plate.

The steel at gage lines 4, 7, 8, 11, 14, and 1 is not greatly heated by the welding processes and there is no reason to suppose that any plastic flow has occurred. It therefore seems reasonable to suppose that the strain at these gage lines is a direct and fairly accurate measure of the stress that has been induced by welding. The average values of the stress for these six gage lines, based upon the assumption that the deformation is elastic and that $E = 30\ 000\ 000$ lb. per sq. in., are 6480 and 11 400 lb. per sq. in. for the section at the bottom of the top panel and at the bottom of the bottom panel, respectively, for the north column, and 8040 and 10 400 lb. per sq. in. for the corresponding sections of the south column.

A temperature stress due to welding varying from 6000 to 11 000 lb. per sq. in. should be given serious consideration, for it is probable that the maximum stress, the sum of the stresses due to dead load, live load, impact, and temperature changes, is greater at some points for the reinforced columns than it was for the original column. Whether or not reinforcing the columns by welding on additional plates has been beneficial depends upon the answer to the question,

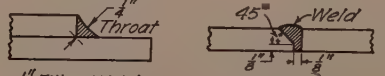
Schedule of Welding Procedure

Columns B1 and B2

1.-Clamp plates P3 & P4 to flanges of Z-bars, in exact position, and tack by one inch of weld at intervals of 18 inches.
2.-Butt weld P4, near side, at top of column.
3.-Fillet weld P4, near side, about 18 inches, laying bead continuously from top down, first on one edge, then on the other edge.
4.-Butt weld P4, far side, at top of column.
5.-Fillet weld P4, far side, same as (3).
6.-Continue fillet weld for P4, alternating near side and far side, as above, until entire length of plate is welded.
7.-Butt weld between P4 and P3.
8.-Fillet weld P3, same procedure as with P4.
9.-Butt weld P3 at bottom of column.

Columns C1 and C2

1.-Clamp plates P5 and P6 in exact position on column and tack by one inch of weld at intervals of 3 feet.
2.-Fillet weld P5, near side, at bottom of column first one edge then the other, laying bead continuously from bottom up.
3.-Make intermittent, staggered welds and shoe-string welds, near side, for about 3 feet, laying bead continuously from bottom up.
4.-Make fillet welds, far side, same as (3).
5.-Repeat (3) & (4) alternating near side and far side to end of plates.
6.-Butt weld between P5 and P6.
7.-Fillet weld P6 same as P5.
8.-Butt weld P5 at bottom of column.



¼" Fillet Weld on B1 & B2 — Bevel and Butt Weld

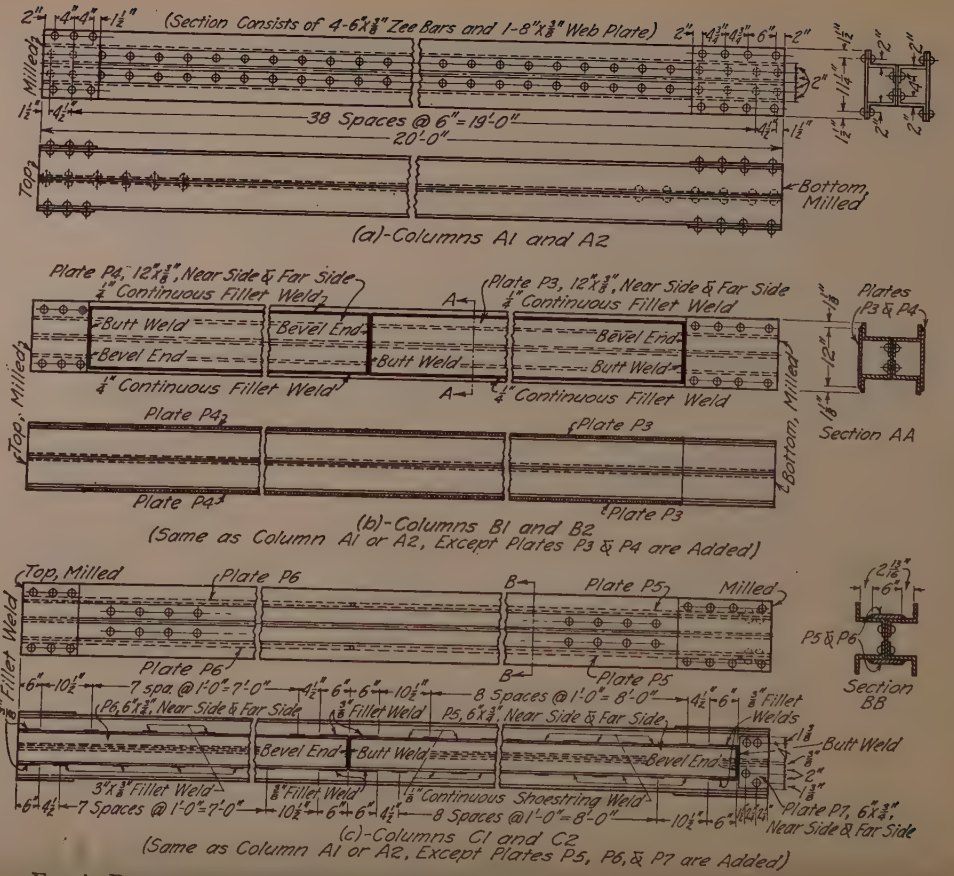

(Section Consists of 4-6"x⅜" Zee Bars and 1-8"x⅜" Web Plate)
Milled
38 Spaces @ 6" = 19'-0"
20'-0"
Top
Bottom, Milled
(a)-Columns A1 and A2
Plate P4, 12"x⅜", Near Side & Far Side
¼" Continuous Fillet Weld
Butt Weld
Bevel End
Plate P3, 12"x⅜", Near Side & Far Side
¼" Continuous Fillet Weld
Plates P3 & P4
Section AA
Top, Milled
Bottom, Milled
Plate P4
Plate P3
(b)-Columns B1 and B2
(Same as Column A1 or A2, Except Plates P3 & P4 are Added)
Top, Milled
Plate P6
Plate P5
Milled
⅜" Fillet Weld
7 spa @ 1'-0" = 7'-0"
8 Spaces @ 1'-0" = 8'-0"
(P6, 6"x⅜", Near Side & Far Side)
(P5, 6"x⅜", Near Side & Far Side)
⅜" Fillet Welds
Butt Weld
3"x⅜" Fillet Weld
⅜" Fillet Weld
¼" Continuous Shoestring Weld
7 Spaces @ 1'-0" = 7'-0"
8 Spaces @ 1'-0" = 8'-0"
Plate P7, 6"x⅜", Near Side & Far Side
Section BB
P5 & P6
(c)-Columns C1 and C2
(Same as Column A1 or A2, Except Plates P5, P6, & P7 are Added)

FIG. 4. DETAILS OF LABORATORY SPECIMENS

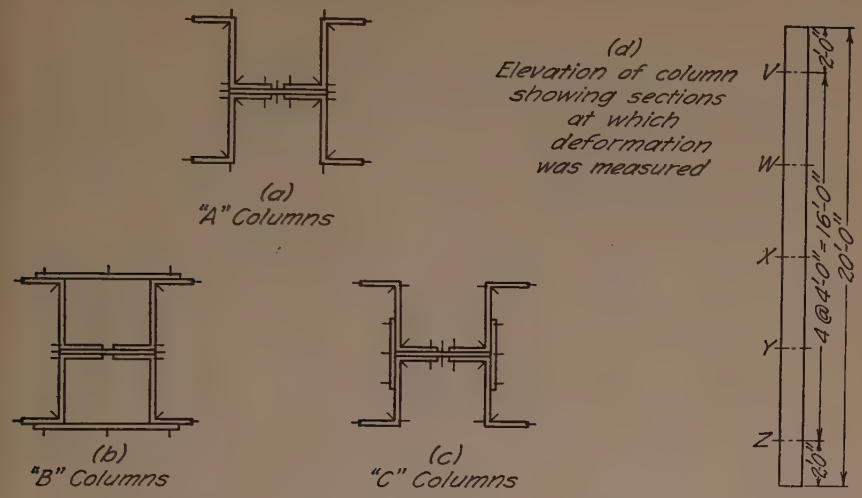


FIG. 5. LOCATION OF GAGE LINES ON WHICH DEFORMATION WAS MEASURED

Is the load-carrying capacity of a column affected by the fact that there are residual longitudinal stresses in the column which are of considerable magnitude and which are compression over some portions and tension over other portions of a section? The tests described in the following chapter were planned to throw some light upon this question.

III. LABORATORY TESTS

7. *Description of Specimens and Tests.*—Six specimens were used in the tests to determine the effect of residual longitudinal stresses upon the load-carrying capacity of steel columns. All six columns, as originally fabricated by riveting, had the same section as the original columns of the Girard viaduct. Each column was 20 ft. long and its ends were accurately machined. Two columns, A1 and A2, which were duplicates, were tested without being reinforced. These are shown in Fig. 4a. Two other columns, B1 and B2, originally duplicates of A1 and A2, were reinforced in the same manner as the columns of the Girard viaduct. These are shown in Fig. 4b. The remaining two columns, C1 and C2, were reinforced in the same manner as the columns of the Swanville viaduct. These are shown in Fig. 4c. The columns were in the vertical position when the welding was done and the welding procedure given on Fig. 4 was followed.

Strain readings were taken on the columns after the riveting had

TABLE 1
PHYSICAL PROPERTIES OF STEEL

Column No.	Yield Point of Steel, in 1000 lb. per sq. in.						Total Yield-Point Strength of Column in 1000 lb.‡
	Z Bars†				Web* Plate	Reinforcing* Plate	
	1	2	3	4			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
A1	36.0	37.2	34.8	36.6	35.5	None	750.5
A2	36.9	35.9	34.1	36.5	35.4	None	740.8
B1	35.9	35.3	34.5	38.3	36.2	34.1	1044.9
B2	35.2	36.0	35.7	35.0	34.8	34.7	1037.7
C1	35.0	34.8	35.0	35.7	36.0	36.9	1069.0
C2	34.7	37.0	35.3	35.1	35.7	36.9	1075.0

*Tension tests.
†Compression tests.
‡Computed from yield point of control specimens.

been completed and after the reinforcing plates had been clamped in place, but before the welding was begun. A second set of readings was taken after welding had been completed and the columns had cooled to the temperature of the air. The difference between the two readings on a gage line, one taken before and the other after welding, is considered to be the deformation due to welding. The location of the sections at which deformations were measured, and the location of the gage lines on a section, are shown in Fig. 5.

All columns were tested in the 3 000 000-lb. testing machine in the Materials Testing Laboratory at the University of Illinois. Spherical blocks were used at both the top and the bottom. The procedure in mounting a specimen was as follows: The top surface of the bottom spherical block was leveled and the block fixed in position by means of steel wedges. The specimen was then centered on the block and allowed to stand free. The wedges holding the bottom spherical block were then adjusted until the top end of the column was centered relative to the top spherical block. This adjustment was very slight for all columns, indicating that the ends had been milled accurately. The upper head of the testing machine was then lowered until the specimen carried a small load, about 20 000 lb. This fixed the position of the upper spherical block so that it bore over the entire upper end of the specimen. The upper block was then fixed in position by means of steel wedges. By this procedure uniform bearing was obtained on both ends of the column but the ends were fixed during the test. Strain readings were taken at the points indicated in Fig. 5 at various load

TABLE 2
COMPARISON OF YIELD POINTS IN TENSION AND COMPRESSION FOR Z BARS

Z Bar No.	Yield Point, lb. per sq. in.	
	Tension	Compression
A22	35 700	35 900
A23	36 500	34 100
B13	36 500	35 000
B14	39 600	38 300

increments. The lateral deflection of the two diagonally-opposite corners of the mid-section of each column was measured from four wires attached at the top and bottom. Readings were taken with a steel scale graduated to 0.01 in.

The Z bars for all specimens were taken from the same heat, likewise all web plates were from the same heat. The material was originally cut 22 ft. long and a 2-ft. length was cut from each piece to be used as a control specimen for determining the physical properties of the material. Three standard tension specimens were cut from each web plate and the yield point of the material for the web as indicated by the drop of the beam, given in column 6 of Table 1, is the average for the three specimens. Sections of the Z bars 4 in. long were machined parallel on the two ends and the yield point of the material, as determined by the drop of the beam, was determined by compression tests. The values for the four Z bars of each specimen are given in columns 2 to 5 of Table 1.

The control specimens for the reinforcing plates were cut from the columns after they had been tested. These control specimens were selected from portions which, as indicated by the strain readings, had not been stressed much beyond the yield point in the tests of the columns. There is, however, a possibility that the yield point of the material may have been increased slightly by the flow of the material, even though the flow was very small. The control specimens for the 6 in. by ¾ in. plates were 6 in. long and 5 in. wide, the outer ½ in. on each edge having been machined away so as to eliminate the material whose properties may have been affected by the welding process. These specimens were tested in compression. The control specimens for the 12 in. by ⅜ in. plates were standard tension specimens. In order to compare the yield point in tension and compression, four standard tension control specimens, one from each flange and two

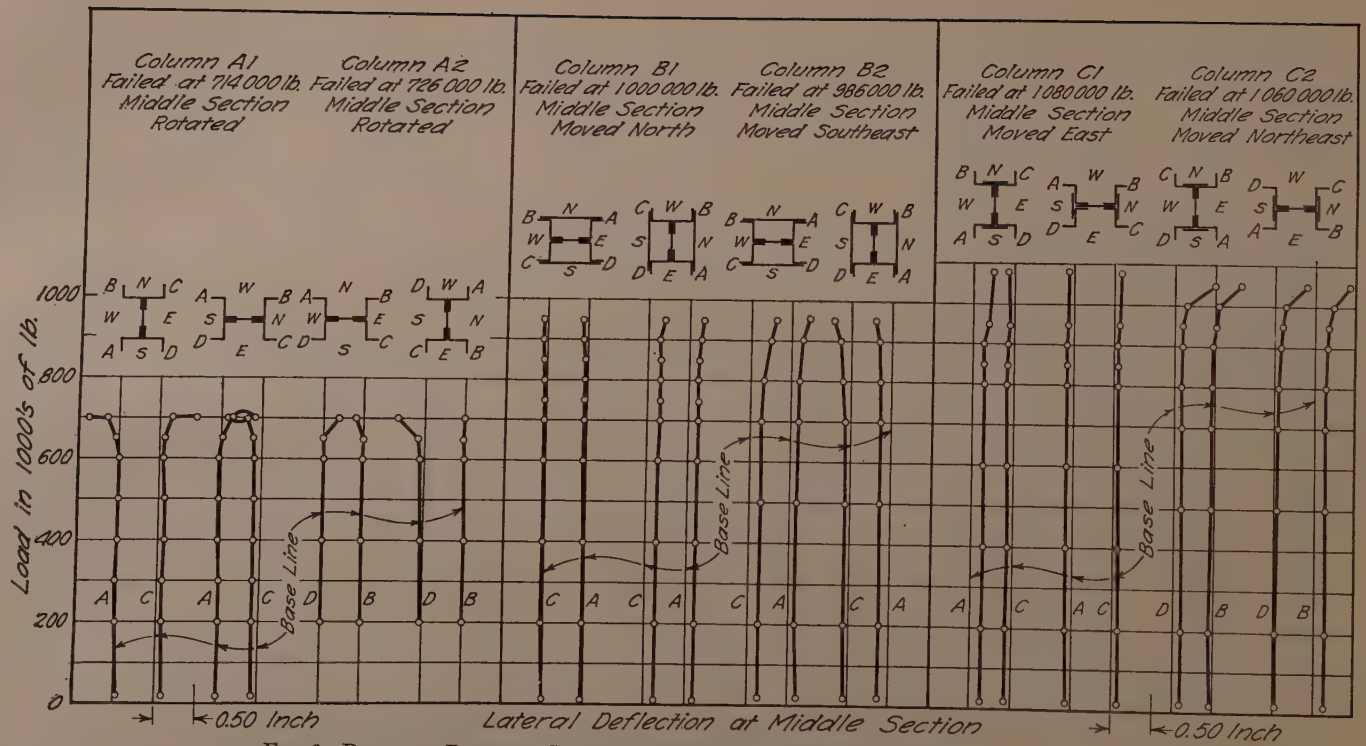


FIG. 6. RELATION BETWEEN LOAD AND LATERAL DEFLECTION AT MIDDLE OF COLUMN

FIG. 7. COLUMNS AFTER FAILURE

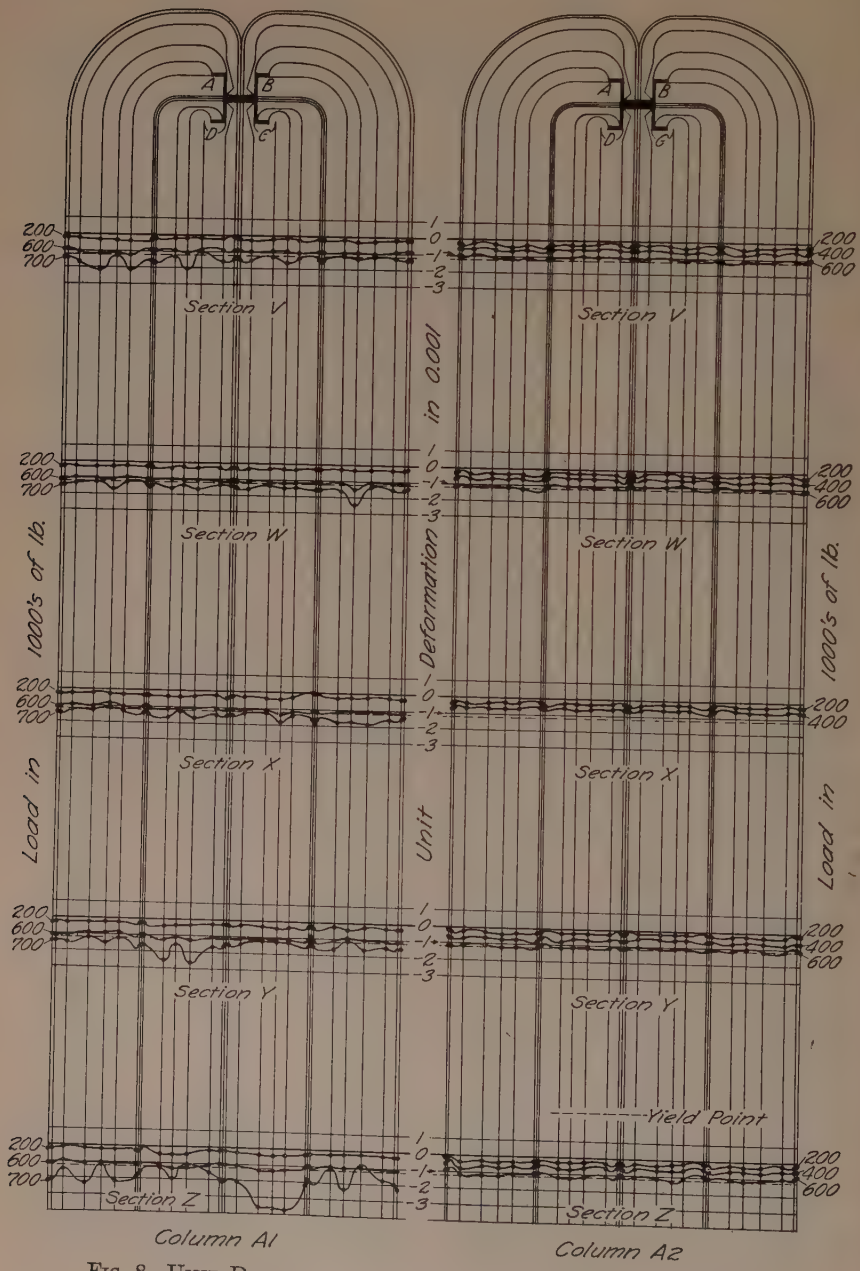


FIG. 8. UNIT DEFORMATION DUE TO LOAD; COLUMNS A1 AND A2

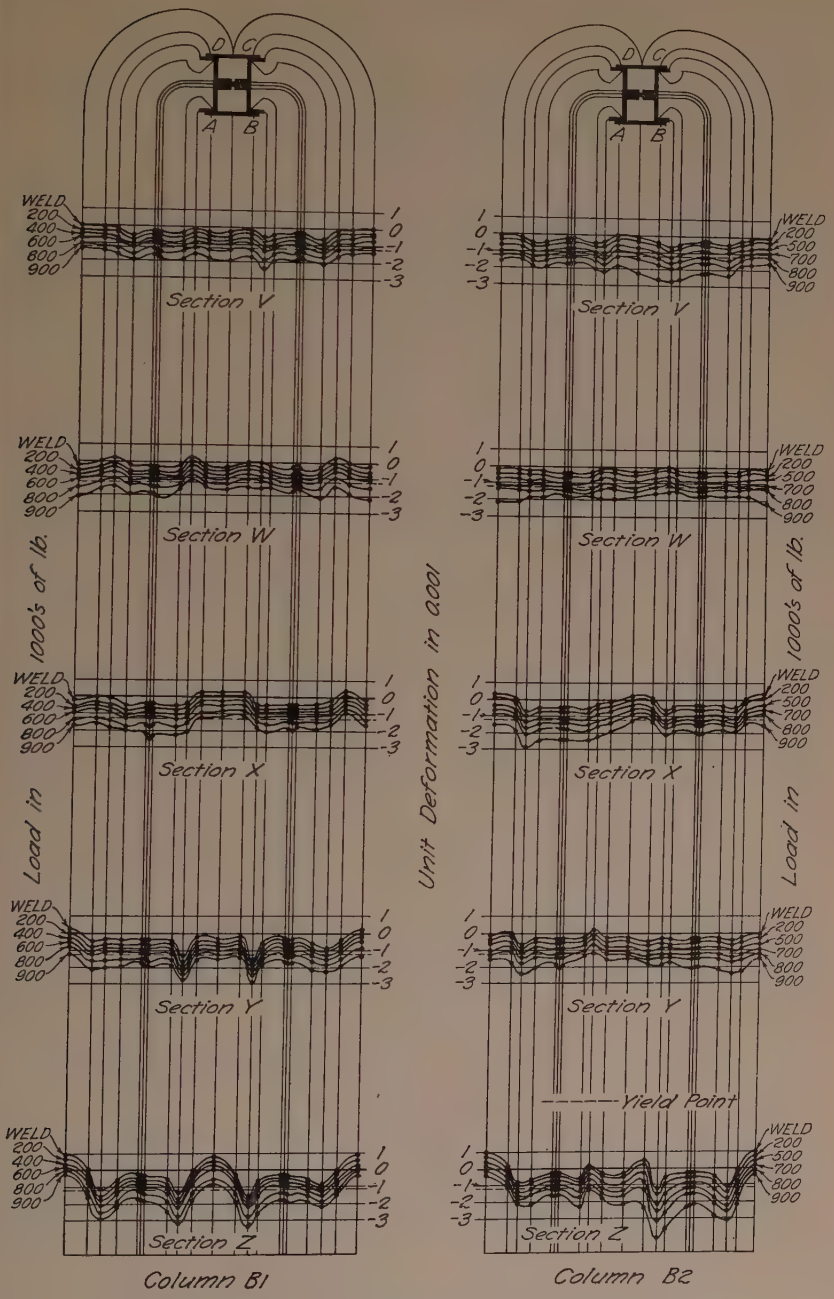


FIG. 9. UNIT DEFORMATION DUE TO WELDING AND TO LOAD; COLUMNS B1 AND B2

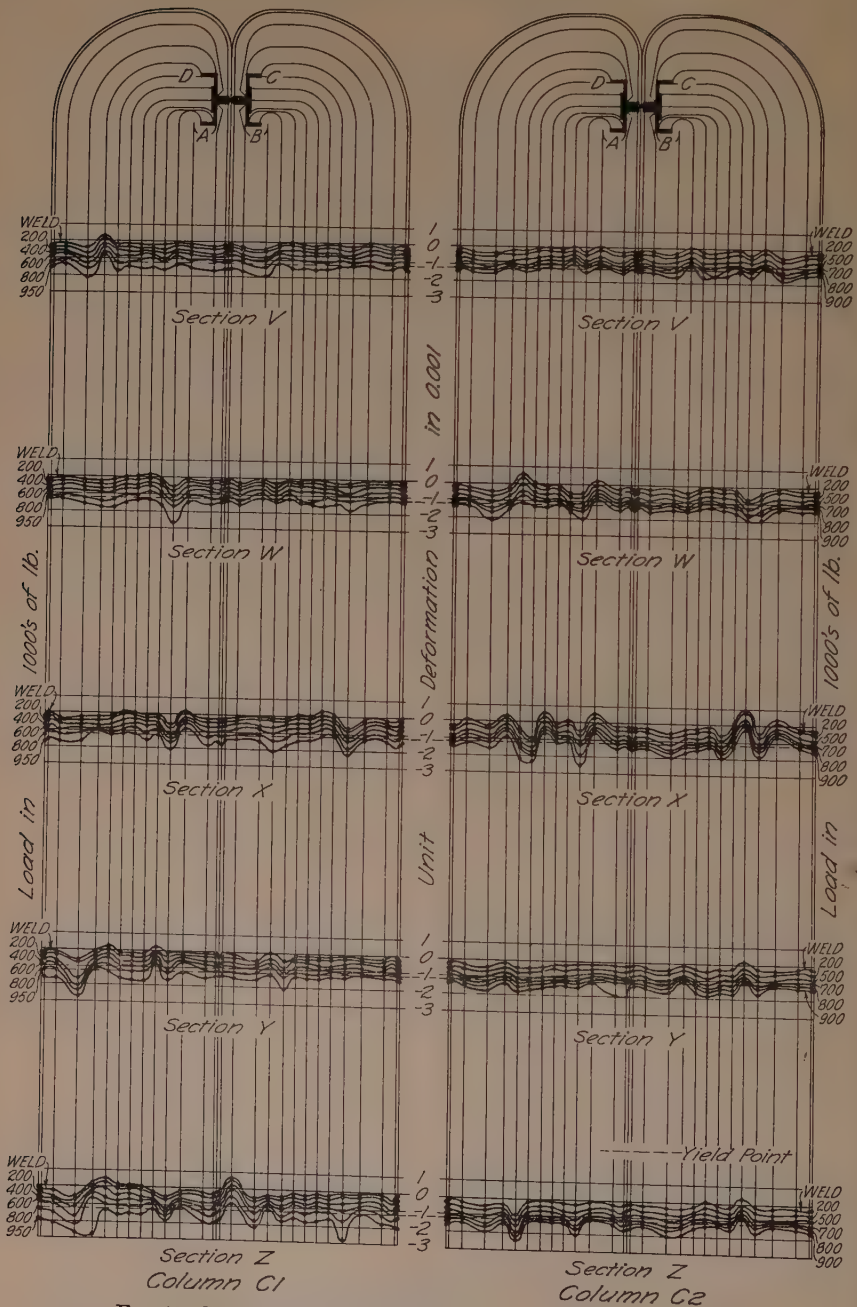


Fig. 10. Unit Deformation Due to Welding and to Load; Columns C1 and C2

from the web, were cut from each of four Z bars. The yield points in tension and compression for these Z bars are compared in Table 2.

The values of the yield point for the columns, taken as the sum of the yield points of the various members comprising a specimen, are given in column 8 of Table 1.

8. *Results of Tests.*—The same procedure was followed in the tests of all columns. The quantities measured include the lateral deflection of the column at the middle and the longitudinal deformation of the steel at the gage lines indicated in Fig. 5. The first readings were taken at a load of approximately 20 000 lb. In general, readings were taken at five or six loads so distributed as to bring the last load just under the estimated ultimate.

The lateral deflection at the middle of the column is shown in Fig. 6. These diagrams show that the columns were originally straight, and that no appreciable deflection occurred until the load was near the ultimate. Figure 7 shows the columns as they appeared after they had been tested.

The variation over the section of the deformation due to welding and due to load is shown for each of the five sections *V*, *W*, *X*, *Y* and *Z* in Figs. 8, 9 and 10. These diagrams indicate that thermal deformation due to welding as great as the elastic deformation corresponding to a stress of 10 000 to 15 000 lb. per sq. in. was quite common, and that, in a few instances, the thermal deformation was nearly or quite equal to the elastic deformation corresponding to the yield-point stress of the material. The thermal deformation appears to be somewhat greater in the B than in the C columns. Moreover, the high thermal stress in the outstanding leg of the Z bars of the B columns might be considered more undesirable than any that occurred in the C columns since there is some probability that the initial failure of the B columns was the wrinkling of the outstanding leg of the Z bar.

The relation between the load and the average unit deformation over the section is shown in Figs. 11, 12 and 13. For each column there is one diagram showing this relation for each of the five sections and one diagram showing the average of the values for these sections. The broken lines show the computed elastic deformation corresponding to the average stress, and the full lines the average of the measured strain.

The measured and computed values of the deformation are in fair agreement at the smaller loads, but the measured values greatly ex-

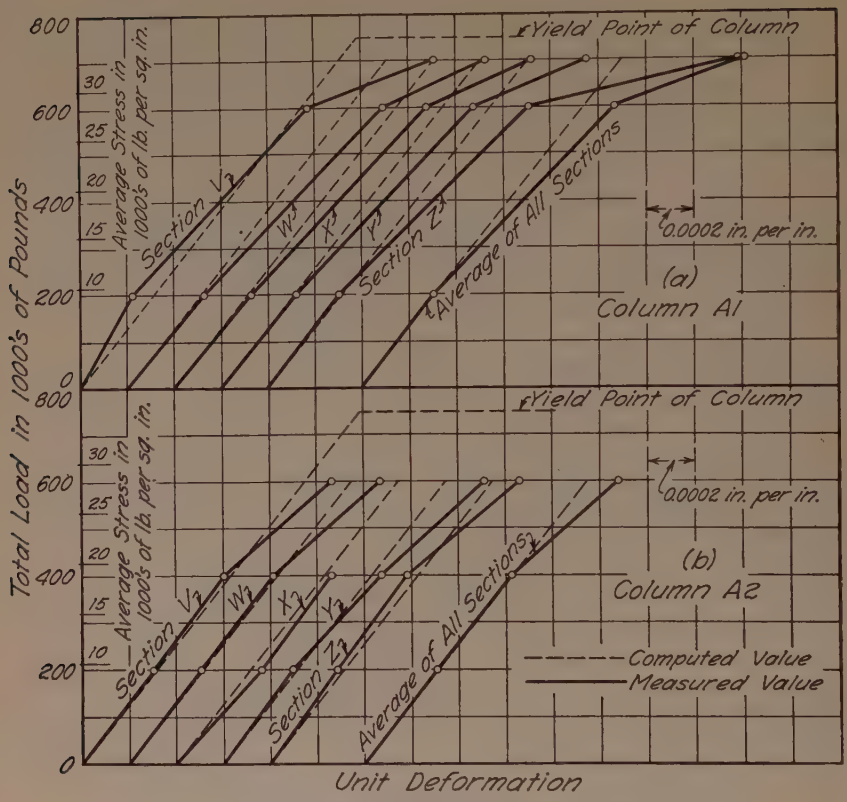


Fig. 11. Relation Between Load and Unit Deformation; Columns A1 and A2

ceed the computed values at loads near the ultimate. There are a number of causes that might contribute to this difference, of which the following are of interest: (1) The presence of experimental errors; (2) the fact that the average deformation on a finite number of gage lines (20 to 28 per section) is not necessarily the average for the whole section; (3) the fact that stress may not have been proportional to strain over a part or all of a section.

Deformation of steel within the proportional limit cannot be measured with a high degree of accuracy and some experimental errors are to be expected. But the fact that the curves for the five sections of each column are similar makes it appear reasonable to assume that the average curve for each column gives, with a fair degree of accuracy, the relation between the load and the average unit deformation for the column.

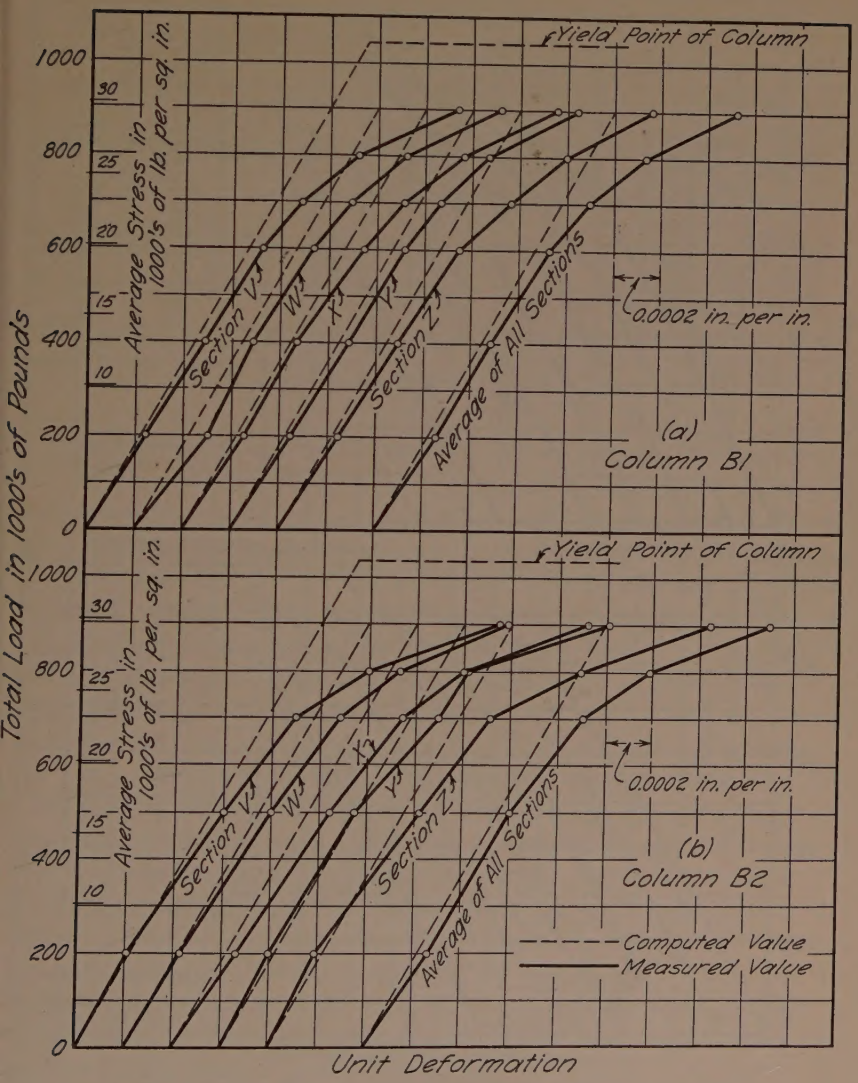


FIG. 12. RELATION BETWEEN LOAD AND UNIT DEFORMATION; COLUMNS B1 AND B2

The deformation was read on 28 gage lines on each of the five sections of columns A1, A2, C1 and C2, and on 20 gage lines on each of the five sections of columns B1 and B2. With so large a number of gage lines, and with gage lines as well distributed as these, especially for the A and C columns, it is reasonable to suppose that the

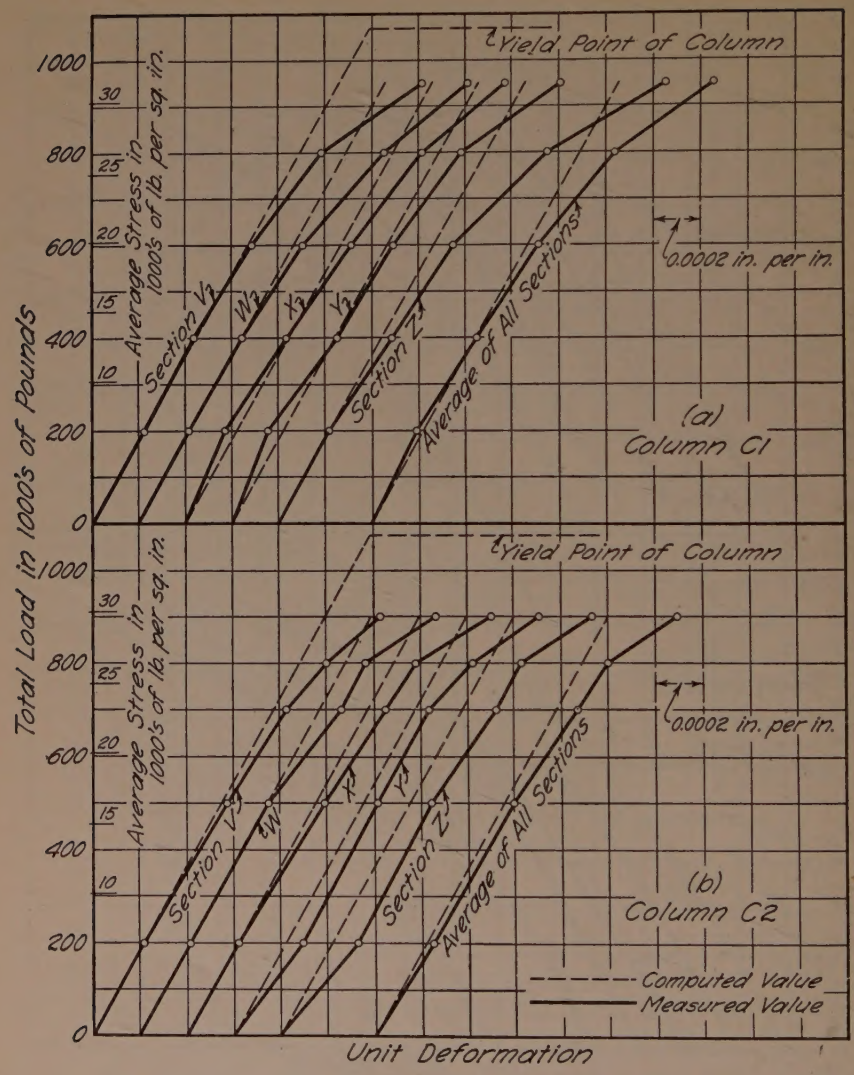


FIG. 13. RELATION BETWEEN LOAD AND UNIT DEFORMATION; COLUMNS C1 AND C2

true average deformation does not vary greatly from the average on the large but finite number of gage lines on which readings were taken.

If a portion of the area of a section is stressed beyond the *proportional limit,* the average strain over the whole section will be greater than the elastic strain corresponding to the average stress. If some

TABLE 3
COMPARISON OF LOAD-CARRYING CAPACITY AND YIELD-POINT STRENGTH OF COLUMNS

Column No.	Area of Section sq. in.	$\frac{l}{r}$	Total Maximum Load in 1000 lb.	Unit Maximum Load in 1000 lb. per sq. in.	Yield-Point Strength in 1000 lb.*	Ratio Total Load to Yield-Point Strength
A1	21.36	66	714.0	33.4	750.5	0.95
A2	21.36	66	726.0	34.0	740.8	0.98
B1	30.36	64	1000.0	33.0	1044.9	0.96
B2	30.36	64	986.0	32.5	1037.7	0.95
C1	30.36	75	1080.0	35.6	1069.0	1.01
C2	30.36	75	1060.0	34.9	1075.0	0.99

*Taken from Table 1. Equals the sum of the yield-point strengths of the various parts as determined from tests of control specimens.

portion is stressed beyond the *yield* point the increase in average strain will be even greater. The departure of the full lines from the broken lines in Figs. 11, 12 and 13 is evidently due to the fact that some part of the section is stressed to the yield point, even though the average stress for the section is less than this limit. For all sections of the A columns, those without reinforcing plates, the full line departed a considerable amount from the broken line at a total load of 600 000 lb., about 80 per cent of the yield point as given in Table 1. A review of the strain gage readings reveals the fact that for all sections one-third or more of the gage lines had deformations in excess of the elastic deformation at the proportional limit at this load. The departure of the full lines from the broken lines was greater for the reinforced than for the unreinforced columns. This is attributed to the fact that the deformation over the section varied more for the reinforced columns than for those that were not reinforced (due to the thermal stresses) and some portions of a section were stressed beyond the yield point at a lower average stress for the former than for the latter. This variation in the unit deformation is apparent in Figs. 8, 9, and 10. The deviation of the broken lines from the full lines was greater for the B than for the C columns.

The ultimate load-carrying capacity of the columns is compared with the yield-point strength in Table 3. The ratio of the former to the latter is given in the right-hand column of the table. The unit load-carrying capacity was slightly greater for the C than for either the B or the A columns. The middle portion of the A columns, those that were not reinforced, rotated about a vertical axis indicating that all portions of the column did not act as a unit. This local failure may have caused the A columns to fail at a lower unit stress than the

C columns. The weakest of the six columns developed 95 per cent of the yield-point strength of the material as determined by tests of control specimens.

It should be noted that the large thermal stresses did not destroy the alignment of the columns, and that this excellent showing of the reinforced columns was made by specimens that were straight.

9. *Summary of Conclusions.*—The tests described in this bulletin apparently support the following conclusions:

(1) If structural steel members are reinforced by welding longitudinal plates to the original section, longitudinal thermal stresses will be produced. The thermal stresses will usually be of the order of from 5000 to 15 000 lb. per sq. in., and in some instances may equal the yield-point strength of the material, which, for the columns tested, was approximately 36 000 lb. per sq. in.

(2) If a column that has been reinforced by welding longitudinal plates to the original section is straight after the welding has been completed, the unit load-carrying capacity will be as great for the reinforced as for the original column. The tests were made on square-ended columns having an l/r ratio of about 65, and this ratio was not greatly affected by the change in section.

(3) The columns reinforced with 6 in. by ¾ in. plates were a little stronger than the ones reinforced with 12 in. by ⅜ in. plates.